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Handbook of Research on Seismic Assessment and Rehabilitation of Historic Structures

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Chapter 20

Seismic Assessment via EC8 of Modern Heritage Structures: Knowledge of the Structure and Analysis Methodologies

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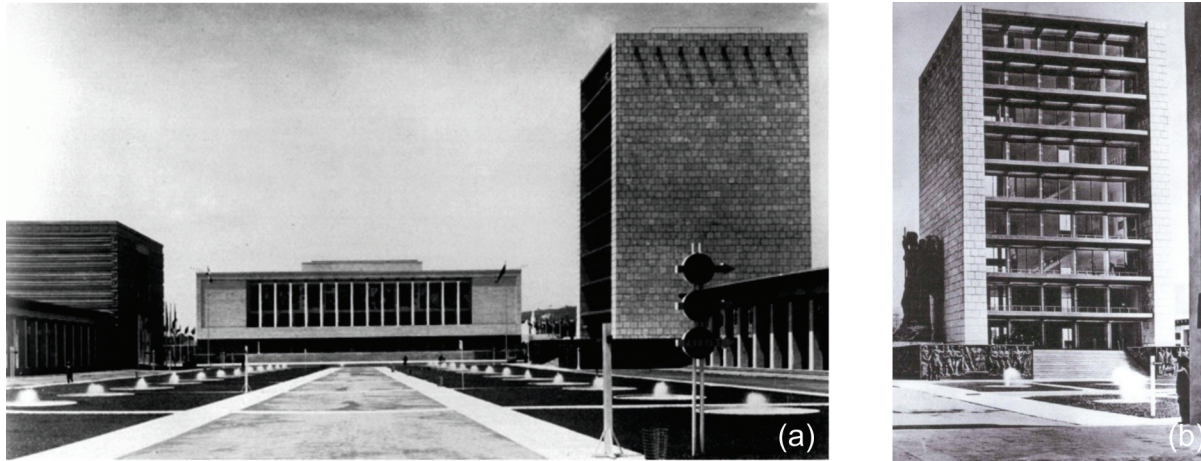
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ABSTRACT

Given the interest earned recently by modern heritage structures, seismic assessment criteria of Eurocode 8, for ordinary reinforced concrete structures, are applied to a modern heritage RC building. The case study, the Tower of the Nations in Naples, allows a discussion on knowledge approaches, analysis methodologies and modeling choices that can be considered. Modal dynamic identification, in situ inspections, and testing provided the necessary knowledge of the structure. Linear and nonlinear models of the structure are built up accounting for tuff infills' stiffness and strength contribution. Numerical modal properties are compared with those obtained through dynamic identification. Lumped plasticity model for reinforced concrete elements and equivalent strut macro models for tuff and concrete infills are employed for the nonlinear model of the structure. Seismic assessment through nonlinear dynamic analyses is carried out for two Limit States. Finally, fragility curves through cloud analysis are obtained for the different limit states considered.

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Figure 1. Mostra d'Oltremare complex (a), Tower of the Nations (b)



INTRODUCTION

The Tower of the Nations is a modern heritage structure, located within the Mostra d'Oltremare urban park in Naples (Siola, 1990). The whole urban park was nominated in 2005 for the inscription in the UNESCO Modern Heritage List (<http://whc.unesco.org/en/events/247/>), given its relevant expression of the cultural and technical background at the time of design and construction. The Tower of the Nations was designed in 1938 by the architect Venturino Ventura, with the help of Carlo Cestelli Guidi (one of the most important structural engineers at the time) and of Guido Quaroni (architect and painter). The building was completed in 1940. The Tower has two glazed façades without any masonry infill (Figure 1a), and it has the other two façades fully infilled by tuff masonry, covered by white travertine plates, see Figure 1b. In 1940, the building had a basement decorated with low reliefs and a big statue representing the Fascist Victory (see Figure 1b); the provisional chalk version of the low reliefs was destroyed during the Second World War.

In the years following its construction, the structure was left to the carelessness. Recently the Tower has been included in a refurbishing project of the entire Mostra d'Oltremare urban park. The project includes the assessment and retrofitting of the Tower.

An example of Eurocode 8 (EC8) based assessment (CEN, 2005) carried out for this modern heritage reinforced concrete (RC) building is herein provided. Notwithstanding the fact that general assessment principles for ordinary RC structures can be easily applied to modern heritage buildings; this kind of buildings can often require specific approaches for structural knowledge, significant modeling efforts, and accurate analysis methodologies.

Therefore, all the assessment steps are reviewed considering the Tower of the Nation as test-bed. Structural knowledge (geometry, details, and materials), dynamic identification, choice of the structural analysis methodology are considered, and specific choices made for the assessment of this modern heritage structure are described. In particular, knowledge of the structure and dynamic identification phases emphasize the role that infills can play in the structural behavior of the building. Thus, infill structural contribution is included in the linear and nonlinear numerical model of the structure. While in the case of

ordinary structures the explicit modeling of infill is a best-practice option, in the case of modern heritage it can be a key issue to capture modal properties and to model nonlinear behavior. Finally, the choice of nonlinear dynamic analysis as analysis methodology for EC8 based assessment allows also carrying out fragility curves for the limit states considered. On the other hand, it is also emphasized how code-based nonlinear dynamic analysis not always leads to reliable fragility curves. In fact, fragility curves can be considered reliable only in the case in which demand over capacity ratios (D/C) carried out for the assessment are close to unity.

BACKGROUND

A first definition of the concept of heritage, and, in turn, of heritage buildings was provided in the Charter of Venice (1964), in which it is stressed, not only the value of the message from the past, but also the common responsibility to their safeguard for future generations, (Vecco, 2010). On the other hand, within heritage buildings, a distinction can be made between historical buildings and monumental buildings. The former are those that can be defined as buildings of artistic or cultural value and which can be found in significant numbers; the latter are those truly unique, (Augusti and Ciampoli, 2000).

Generally speaking, one of the criteria classifying heritage or historical structures is that long time has passed since their construction, (Gulkan and Wasti, 2009). On the other hand, *old* is a relative term, and, in practice, can be as low as 50-100 years.

As an example, the definition of architectural heritage is experiencing an evolution in time. In particular, the category of modern heritage structures is earning increasing interest (e.g., Ronca et al. 2009; Mosoarca and Victor, 2013). Hence, more and more reinforced concrete (RC) structures can be classified in this category, although RC is a rather new structural material in comparison with wood and bricks.

In the above framework, seismic assessment of modern heritage structures is an issue that earns interest in seismically prone regions in Europe, such as Italy, Greece, Portugal or Turkey (e.g., Syrmakezis, 2006). The uniqueness characteristic of heritage buildings requires specific assessment approaches (Augusti and Ciampoli, 2000), often ruled by specific guidelines in the different countries (e.g., Ministero dei Beni e le Attività Culturali 2010). Thus, it is rare to find code approaches for ordinary structures employed for the assessment of heritage buildings. On the other hand, the case of modern heritage RC buildings is, in this sense an exception. In fact, seismic assessment provisions meant for ordinary structures can be to some extent applied to this category of building (see also De Luca et al., 2014).

Seismic Assessment via EC8 of a Modern Heritage Structures

EC8 seismic assessment procedure is analyzed step-by-step emphasizing the aspects on which the case of heritage buildings requires specific attention or additional procedures (e.g., dynamic identification). Each assessment step is described in the following providing a critical review and employing the Tower of the Nations as test-bed. It is worth noting that the main target herein is the outline of the process and the analysis of specific methodological steps; so in most cases details on the case study are not always provided unless functional for the methodological review. Further details on the test-bed case study building can be found in De Luca et al. (2014), Ranieri et al. (2013), and Ranieri and Fabbrocino (2011).

Structural Knowledge

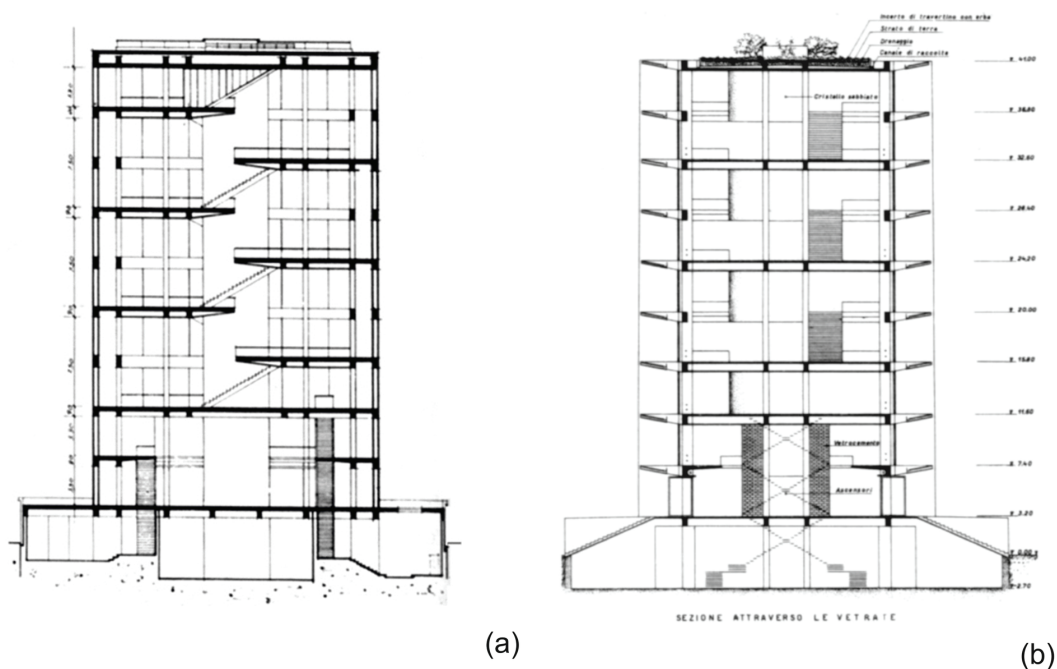
Input data for the assessment of existing structures shall be collected from a variety of sources, including: available design documentation of the building, relevant generic data sources (e.g. codes and standards at the time of construction), in situ investigations, and laboratory tests. EC8 provides a quantitative framework for the definition of knowledge levels of structures in order to choose the admissible type of analysis and the appropriate confidence factor to be employed in the assessment (CEN, 2005).

The quantitative framework provided by EC8 for ordinary RC structures is a suitable reference for RC heritage buildings. On the other hand, for heritage buildings, the number of investigations and tests is subjected to different criteria. The organization of investigations and test campaign has to consider whether it is possible to approach with destructive techniques or not. Even if a limited number of destructive investigations and tests can be performed, it is likely that they are not going to meet specific quantitative prescriptions provided by the code for ordinary RC structures.

As an example for the Tower of the Nations, it was possible to perform destructive investigations and tests, and the knowledge level achieved can be classified as a *normal* knowledge level (KL2) for the structure, according to EC8. Still, the choice of KL2 passes through the expert judgment of the operators, since a heritage structure can be seldom classified according to code criteria.

Following the guidelines suggested by EC8, the whole geometry of the structural system was determined for the Tower of the Nations. In situ inspections and simulated design procedure, aimed at determining reinforcements in beams and columns, were employed for the definition of structural details. Geometry was retrieved from the original architectural drawings, and thanks to a structural survey, see Figure 2. The Tower of the Nations is characterized by ten storeys, 44.00 meters tall, and it has a roughly square plan (Figure 2). It is 23.50 m long in North-South direction (in the following Y direction), and

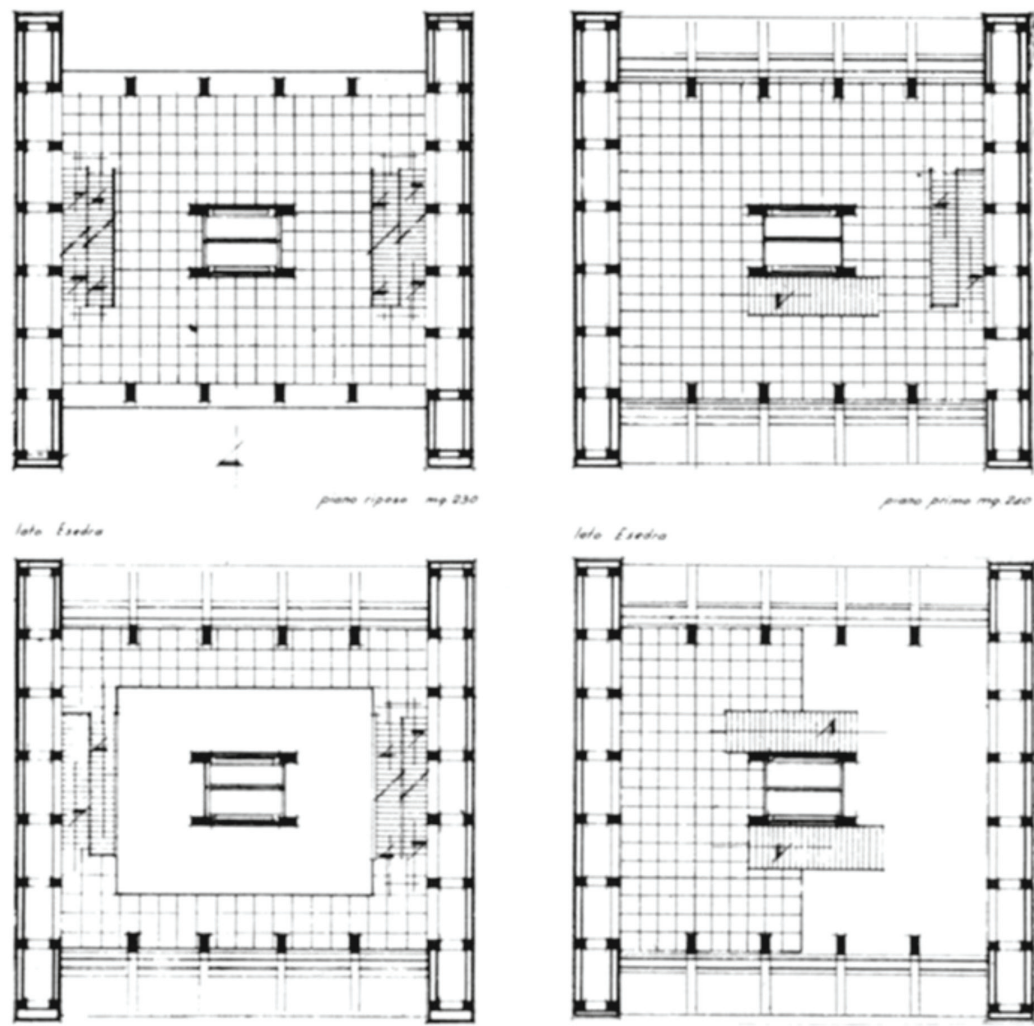
Figure 2. Original drawings: transversal (a), and longitudinal (b) sections



23.70 m wide in the other direction (X direction), see Figure 3. The structure has a system of staggered floors that occupies only one-half of the plans from level forth to ninth (see Figure 2a, Figure 3). The peculiarities of the structural system show that horizontal forces, probably wind action, were taken into account in the design.

The structural system of the Tower is very innovative considering the age of construction. The building is characterized by a three-dimensional frame system. Two couples of frames (seven bays) are placed at two opposite sides of the plan (along Y-Z plane). The two frames on East and West façades (YZ) are stiffened by a concrete bracing system. Concrete bracings alternatively occupy the bays of the prospect frames from first to eighth level. In this direction (YZ), there are also tuff masonry infills without openings. In X direction, the structural system is composed by frames with different number of bays. Frame

Figure 3. Original drawings: first level plan (top-left); second level plan (down-left); third level plan (top-right); typical plan (down-right)

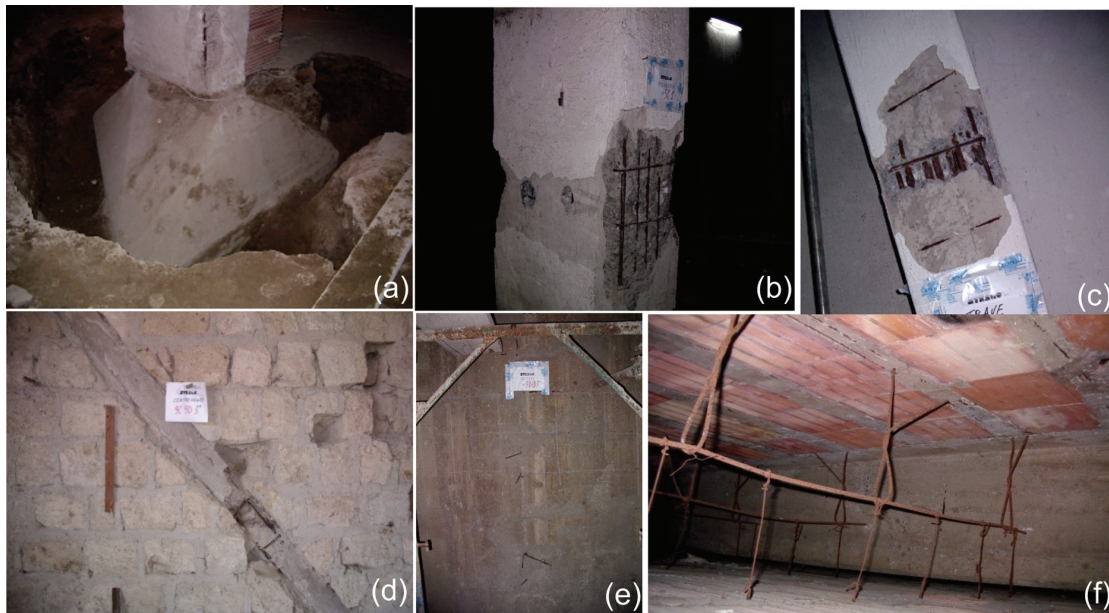


system in this direction is integrated by slim concrete walls situated at the center of the plan (lift-shaft) and at the two sides of the plan. Along X direction there are no masonry infills. The Tower has a concrete basement, 36.50m long in Y direction, and 36.00 m large in X direction (see Figure 2).

Structural details, such as amount and detailing of reinforcement in the RC elements are obtained by in situ inspections (see Figure 4) and through non-destructive methods. Simulated design is carried out aimed at integrating information from inspections for longitudinal reinforcement of beams (Verderame et al., 2010). The role of simulated design in heritage buildings is crucial for the knowledge of the structure. In fact, while in EC8 framework it is considered as a tool of knowledge in the case in which no structural drawings are available, in the case of heritage buildings it becomes a tool for the integration of structural detail definition.

Columns' sections are rectangular; they are oriented with the strong axes along Y direction, and placed in the two principal frames in X direction. Sections change from the values $450 \times 1100 \text{ mm}^2$ at the first storey to $300 \times 800 \text{ mm}^2$ at the tenth storey. In the lift-shaft, rectangular columns have the strong axes in X direction; sections vary from $900 \times 400 \text{ mm}^2$ at first storey, to the $500 \times 400 \text{ mm}^2$ at tenth storey. Concrete walls thickness is 150 mm in the lift-shaft and 100 mm for elements in the lateral zone of the frames in X direction. Beam sections vary from $250 \times 500 \text{ mm}^2$, for elements placed in the seven-bay frames along Y direction, to $300 \times 800 \text{ mm}^2$ for elements that connect the lift-shaft with the lateral seven bay frames in the two directions. Wide beams' sections at the second storey are $500 \times 220 \text{ mm}^2$. One way RC slabs have 180 mm thick joists, and 50 mm of full concrete slab. Stairs are realized with full concrete slabs (230 mm thick). Full concrete slabs (230 mm thick) are also placed close to the stairs and close to the lift-shaft at each staggered floor. The amount of longitudinal reinforcement in columns rarely exceeds

Figure 4. In situ inspections: (a) foundation footing; (b) columns' and (c) beams' reinforcement; (d) concrete diagonal brace in tuff masonry infill; (e) concrete infill; (f) floor slabs (De Luca et al., 2014)



0.7% of the gross section Stirrups in beams and columns are characterized by 6 mm diameter and spaced at 250 mm, see Figure 4b, while shear reinforcement in beams is characterized by both stirrups (Figure 4c) and diagonal reinforcements at the ends of the elements.

Material properties are obtained through both destructive and non-destructive tests. Non-destructive test methods are employed in conjunction with destructive tests in accordance with EC8 provisions. Concrete cylindrical compressive strength (f_{cm}) is evaluated equal to 16MPa from a combination of non-destructive test and from concrete specimens, while average yielding strength (f_{ym}) is equal to 275 MPa. The evaluation of concrete compressive strength is also checked through the comparison with dynamic identification results, as it will be shown in the following.

Dynamic Identification

The practice of taking advantage of system identification to reduce the modelling uncertainties, fairly common for ordinary buildings and bridges, is rapidly spreading also to historical and heritage structures. An increasing number of applications appeared in literature in the last few years. These works emphasized the potentialities of Operational Modal Analysis (OMA) to estimate the modal properties of either selected macro-elements (Conte et al., 2011) or the whole structure (Gentile and Saisi, 2007). In fact, traditional input-output modal analysis techniques, even if well established, are often not recommendable in the case of historical structures because of the input control and test execution problems usually related to the application of an artificial excitation. On the contrary, ambient vibration tests are less expensive and faster than traditional input-output tests and the interference with the regular use of the structure is minimized.

Moreover, the large number of highly sensitive sensors and fully computer-based data acquisition systems available on the market (Jacobsen and Thorhauge, 2009) allow properly resolving the structural response to ambient vibrations, whose wide-band frequency content usually provides excitation for a significant number of normal modes. These characteristics make them more suitable for investigations about the dynamic behavior of historical structures and the refinement of numerical models, see Rainieri et al. (2013) for details.

The application of OMA to the Tower of the Nations allowed to select a reliable modeling approach, and to validate results of in situ inspections and tests. The ambient vibration response of the Tower has been measured at the fourth and the fifth level of the building and at the roof. The roof and the fifth level have been instrumented in two corners. At each corner two force balance accelerometers, measuring in two orthogonal directions, have been placed. Another couple of accelerometers have been placed at the fourth level. The ten accelerometers have been placed directly in contact with the concrete slab and parallel to the main directions of the building, in order to get both translational and torsional modes of the structure. A detailed description of OMA applied to the Tower of the Nations can be found in Rainieri and Fabbrocino (2011) and Rainieri et al. (2013). Figure 5 shows singular value plots and peaks relative to the first six modes are highlighted. Table 1 shows periods' and damping ratios' evaluation for the first three modes of the structure.

Data in Table 1 are employed for different purposes in the EC8 based assessment of the building. First, the three periods are compared with linear model of the building to check the evaluation of material properties assumed for concrete and tuff infills. Secondly, damping ratios are employed as an indicator

Figure 5. Singular value plots and peaks for the first six modes
 Courtesy of Ranieri and Fabbrocino (2011).

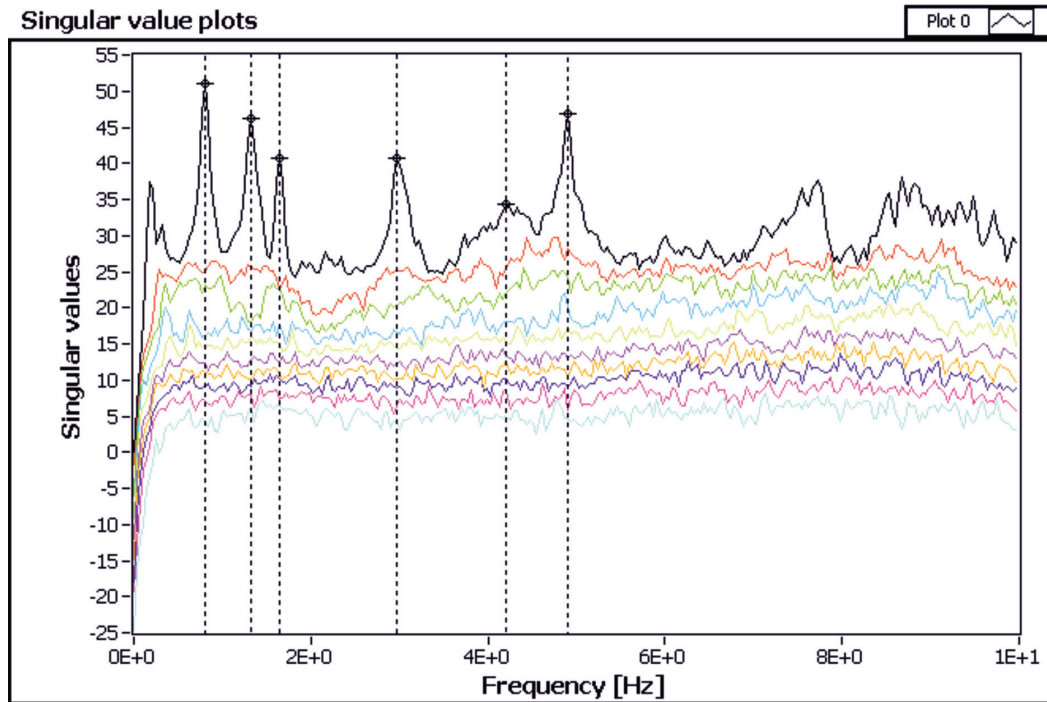


Table 1. Results, average values, of dynamic identification for the Tower of the Nations

Mode Number	Type	Period [s]	Damping Ratio [%]
1	Translation (open side), X	1.25	0.4
2	Translation (blind side), Y	0.75	1.2
3	Torsion	0.60	0.6

(Adapted from Rainieri et al., 2013)

for the choice of damping settings in the nonlinear model of the structure. The employment of dynamic identification results is made through a comparison of results in Table 1 and those obtained through linear modeling of the building.

Linear model of the structure is built up using SAP 2000 (Computer and Structures, 2007). Beams, columns and bracings are modeled as elastic mono-dimensional elements (frame). The numerical characterization of the dynamic response of the Tower was based on the implementation of a number of numerical models. Modal analyses have been carried out through SAP 2000 (Computer and Structures, 2007). Correlation with the experimental results has been evaluated by defining a number of model classes through the combination of the following modelling assumptions:

- Absence vs. presence of tuff masonry walls;
- Absence vs. presence of the basement structure;

- Shell elements vs. rigid diaphragm to model floors.

The different assumptions are investigated because of the main uncertainties affecting the numerical modelling of the structure (see Ranieri et al., 2013). After an optimization with respect to dynamic properties, slabs, concrete infills, and tuff masonry infills, are modeled as shell element characterized by in-plan behavior only (membrane) elements. Stairs are modeled as thick shell. The basement is finally not explicitly modeled, and translational restraints are placed at the first floor, since it has a negligible interaction with the Tower, and linear dynamic behavior is slightly affected by its presence (Rainieri et al., 2013).

Concrete Young modulus (E_c) is assumed equal to 25300 MPa, evaluated as a function of concrete medium compressive strength f_c according to the expression reported in (DM14/01/2008). Tuff infills properties are taken according to the average values reported in (CS.LL.PP., 2009); thus, Young modulus (E_w) is equal to 1260 MPa and shear modulus (G_w) is 0.30 times E_w . Figure 6 shows the two different linear models: the bare model, in which there are no tuff infills (see Figure 6a), and the infilled model, in which infills' structural contribution is accounted through shell elements (see Figure 6b).

Table 2 shows modal properties obtained for bare and infilled model for the first three modes. It is worth noting that tuff infills affect strictly the first translation mode in Y direction (second mode) and the third torsional mode. Linear dynamic properties of the infilled model show a good agreement with experimental results obtained by the dynamic identification made on the building (see Table 1).

Infilled linear model accurately captures periods with errors within 6%, and it shows a very accurate match for the second mode (along Y direction). This result emphasizes the necessity to account for infills explicitly in the structural model of the building for the seismic assessment.

*Figure 6. Geometrical model of the structure without and with tuff infills
(De Luca et al., 2014)*

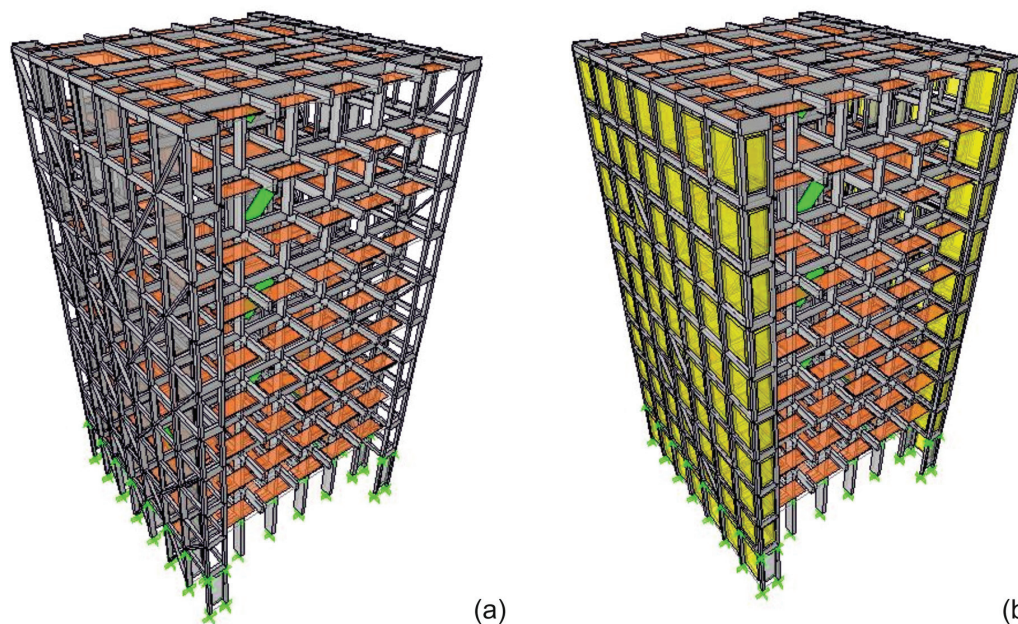


Table 2. Modal properties for bare frame model and the tuff infilled

Model	Mode	T [s]
Bare	1	1.35
	2	1.10
	3	0.93
Infilled	1	1.33
	2	0.74
	3	0.64

(Adapted from De Luca et al., 2014).

ANALYSIS METHODOLOGIES

Analysis methodologies can be classified in two big families according to the approach chosen for the classification. Each family differs because of the basis of the classification: if it is made on the basis of actions or on the basis of response. Thus an analysis method is static or dynamic or it is linear or nonlinear. The above classifications end up in four different analysis methodologies which represent, in turn, the four analysis options provided by most of the recent seismic codes, and also by EC8 part 1 (CEN, 2004). According to Eurocode 8 four analysis methodologies can be employed: (i) linear static analysis, (ii) linear dynamic analysis (i.e., response spectrum analysis), (iii) nonlinear static analysis, (iv) nonlinear dynamic analysis (i.e., nonlinear time-history).

Linear static analysis can be employed when the structure can be considered regular in elevation, according to criteria provided in the code, and if the fundamental period of the structure does not exceeds the value of 2.0 seconds and four times T_D , one of the characteristic periods defined by code spectrum at the site. When employing this analysis method an approximate formulation for the evaluation of the period can be considered (if the height of the structure does not exceeds 40 meters). Such an approximate formulation results in a very conservative evaluation of force demands. Ductility is accounted for by means of the behavior factor q in EC8.

Response spectrum analysis is the routine method for the design of structures according to EC8 part 1 (CEN, 2004). Since compliance to regularity criteria in plan and elevation is not requested, the code prescribes to consider a number of modes so that the sum of the effective modal masses amounts at least 90% of the total mass of the structure, and all modes with effective modal masses greater than 5% are taken into account. Modal combination of the responses can be made according to SRSS rule if the relevant modal responses may be regarded as independent of each other. If the latter condition is not verified CQC rule shall be adopted (see also De Luca and Verderame, 2013a).

Regarding nonlinear static analysis, Eurocode 8 suggests the capacity spectrum method approach provided in Fajfar (1999). At least two vertical distributions of the lateral loads have to be considered (a modal pattern and a uniform pattern). It is worth noting that no restrictions are considered for this kind of analysis, unless the structure has a predominantly torsional first mode of vibration.

In the case of nonlinear time-history analysis sets of three or at least seven accelerograms can be considered if the demand is evaluated as the maximum of the set or as the mean of the sets respectively (see Iervolino et al. 2010a).

In EC8 part 3 (CEN, 2005), for existing buildings, linear analysis (both static and dynamic) are allowed considering the elastic spectral acceleration or the elastic spectrum without the application of any reduction factor and applying these methods according to a displacement-based approach (it means that verification of ductile mechanisms should be based on deformations). These kinds of analyses are subjected to an applicability condition that checks that the demand is uniformly distributed over the structure, allowing the application of equal displacement rule at single element level. The condition is based on the control that the ratio between maximum and minimum D/C results within 2.5, selected among those that exceeds unity. The latter condition emphasizes that in the case of existing building a linear analysis method should be considered as an exception. EC8 part 3 allows also the so called q factor approach, resulting in the same analysis method considered for new design buildings but limiting the value of the q factor to 1.5, unless compliance criteria for local and global ductility according to EC8 part 1 (CEN, 2004) are met. As it is expected, no limitations are provided by EC8 part 1 (CEN, 2004) when nonlinear dynamic analysis is employed. On the other hand, according to EC8 part 3 (CEN, 2005), KL2 has to be achieved for existing buildings if nonlinear analysis methodologies are employed (static or dynamic).

Nonlinear dynamic analysis is chosen for the assessment of the Tower of the Nations. This kind of analysis allows taking into account the complex structural system considered, the effect of higher modes and nonlinear hysteretic behavior of elements and infills. This choice complies with the necessity of a proper characterization of seismic response for heritage buildings, and it will be shown that, in some cases, it also allows carrying out fragility curves for the structure, providing a proper characterization of seismic response that accounts for record-to-record variability.

Seismic input selection is made according to EC8 prescriptions (CEN, 2004). For each damage states considered, two pairs of seven linearly scaled accelerograms are selected through Rexel 3.5 software (Iervolino et al., 2010b), see Figure 7. Spectral compatibility with the two target spectra of the area is considered according to EC8 selection criteria, and limiting the scaling factor up to 2 (Iervolino et al., 2010a).

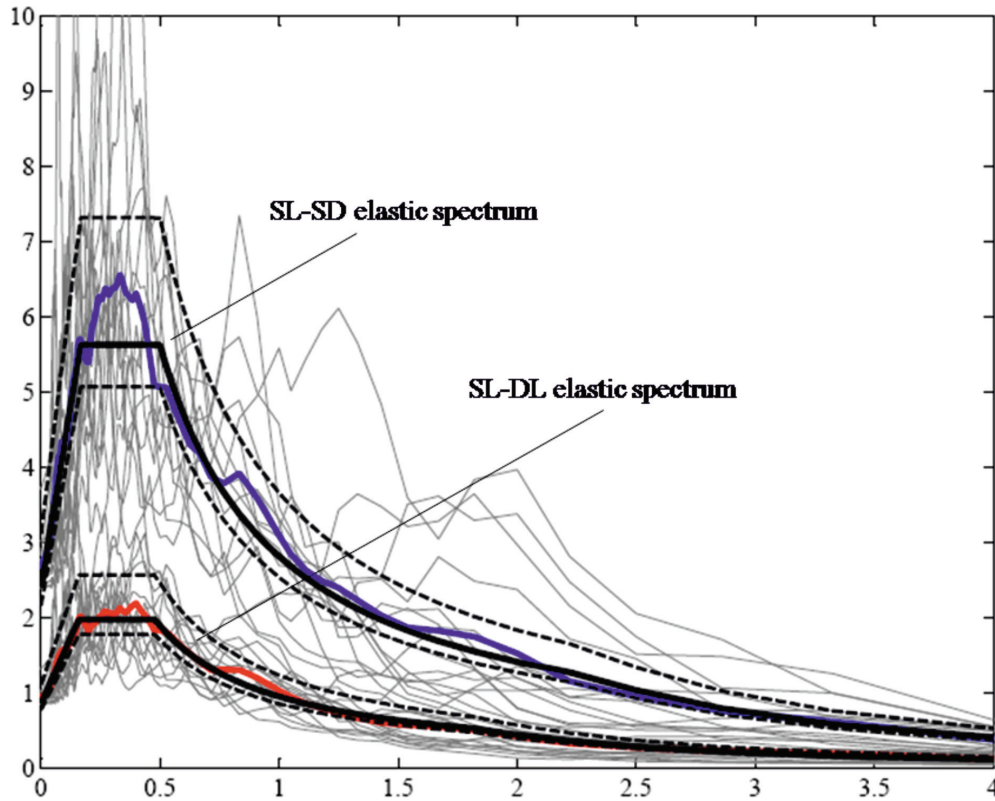
INFILLS' STRUCTURAL CONTRIBUTION

Dynamic identification of the structure emphasized the necessity to account for stiffness infill contribution for the determination of linear properties of the building. This observation suggested considering infill contribution in terms of stiffness and strength also in nonlinear modeling.

According to actual seismic codes, such as the EC8 (CEN, 2004), specific prescriptions are provided regarding structural detailing and masonry infills issue is also considered: in new design buildings infill walls need to be considered in the modeling if they contribute significantly to the lateral stiffness and resistance of the building.

According to EC8, in the design process, infills in principle are still considered as nonstructural elements. Masonry infills role can be even more relevant in ordinary existing structures; that is why the analysis of seismic response of existing RC buildings should take into account the presence of infill elements and interaction between these elements and the primary RC structural system (see also Verderame et al., 2011). Non-structural elements contribution plays a significant role in the assessment, even in the case of modern heritage RC structures in which materials employed are not conventional, e.g., tuff

Figure 7. Sets of records: severe damage (SD) and damage limitation (DL)
(De Luca et al., 2014)

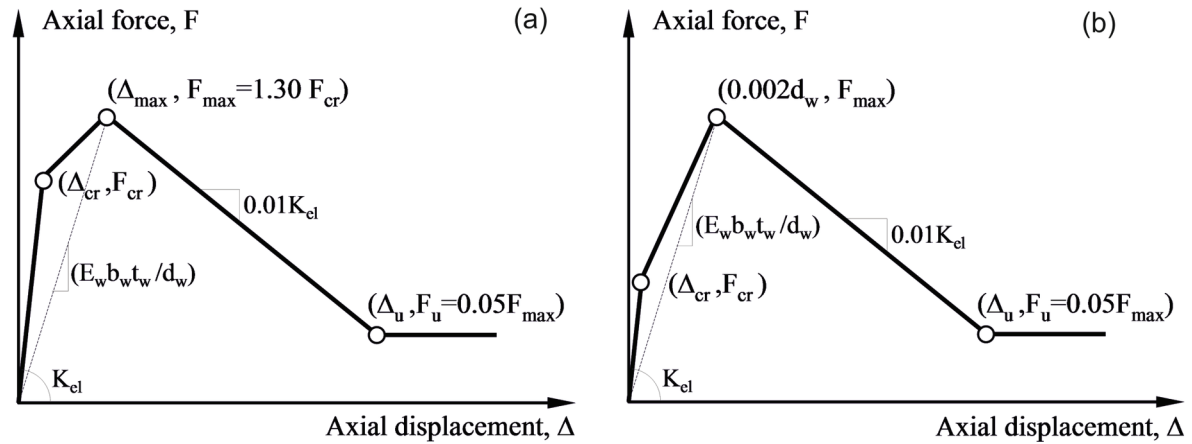


rather than clay hollow bricks. Nonlinear characterization of tuff infills has to take into account specific mechanical properties of this material (Faella et al., 1993; Calderoni et al., 2010, CS.LL.PP., 2009). The analysis is carried out considering for tuff infills G_w equal to 378 MPa and the cracking shear stress (τ_0) equal to 0.04 MPa. In the case study of the Tower of the Nations, tuff and concrete infills are modeled through an equivalent strut macro-model (Fardis and Panagiotakos, 1997). They are two crossing diagonal axial springs that can carry compression only. Tuff infills of the Tower of the Nations are characterized by an unusual shape factor with respect to typical practice in ordinary RC buildings; in fact, h_w is higher than L_w . According to the latter observation, the consistency of the strut mechanism is verified through an empirical formulation, resulting from experimental studies on brick and concrete infilled non-ductile frames (Al-Chaar et al., 2002). In Figure 8a is shown a schematic description of the characterization of strut macro-models employed for tuff masonry.

Due to their poor longitudinal reinforcement and their limited thicknesses (100 mm lateral; 150 mm in the lift-shaft zone), concrete walls are treated as *concrete infills*. The only shear stiffness is considered, and a strut macro-model is assumed, in analogy with that assumed for tuff infills. In Figure 8b is shown a schematic description of the characterization of strut macro-models for concrete infills.

A detailed description of nonlinear modeling of tuff and concrete infills is provided in De Luca et al. (2014).

Figure 8. Force-displacement envelope scheme – (a) tuff infills; (b) concrete infills
(De Luca et al., 2014)



THE TOWER OF THE NATIONS: ASSESSMENT RESULTS

Lumped plasticity modeling approach is employed for the nonlinear model of the Tower. Nonlinear modelling of structural elements (beams, columns, diagonal braces, concrete infills, tuff infills) is carried out using *link* hysteretic elements in SAP2000 (Computer and Structures, 2007). The link element is a nonlinear spring with six independent internal deformations for which a generalized force deformation nonlinear relationship can be defined. In the nonlinear model of the Tower of the Nations, link element has been employed in different ways.

Nonlinear behavior of RC columns is represented by a moment-rotation ($M-\theta$) relationship, evaluated *a priori*. In particular, each $M-\theta$ is defined through four characteristic points: cracking (cr), yielding (y), maximum (max) and ultimate (u). Chord rotations corresponding to each point of the $M-\theta$ relationship are evaluated according to literature or experimental results. Yielding and ultimate chord rotations of columns are evaluated according to formulations provided by EC8 part 3 (CEN, 2005); chord rotation corresponding to maximum moment is evaluated as function of the yielding value, according to the experimental results shown in Verderame et al. (2008a; 2008b), and Di Ludovico et al. (2013). The model assumed for beams is similar to that assumed for columns. A degrading hysteretic relationship in strength and stiffness is assumed for both columns and beams (Dowell et al., 1998).

Brittle failures of columns and beams are excluded *a priori* because shear strengths, evaluated according to EC8 (CEN, 2005), are larger than plastic shear of these elements. Thus, all columns and beams can be classified to have ductile behavior; flexural failure occurs before brittle failure. These results are expected because columns and beams are characterized by very low longitudinal steel ratios that allow favorable shear-flexure hierarchy (De Luca and Verderame, 2013b). Notwithstanding the favorable shear-flexure hierarchy in RC elements, local interaction between infills and RC elements could result in a preemptive brittle failure. This kind of interaction can be captured through multiple strut modeling adopted of the infills (see also Verderame et al., 2011), or considering approximate verification approaches (e.g., ASCE/SEI 41-06, 2007). On the other hand, it should be noted that tuff is characterized by poor mechanical properties and the occurrence of this kind of failure can be excluded when approximate calculations are carried out.

Concrete diagonal braces represent one of the peculiar design issues of the structure and they influence strictly the seismic response of the Tower. Concrete struts are characterized by a $350 \times 150 \text{ mm}^2$ section, lightly reinforced with four 10mm longitudinal bars. Due to the reduced transversal section dimensions of the concrete bracings, and due to their poor longitudinal reinforcement, the only axial behavior of these elements is taken into account and modeled. Axial springs of each concrete diagonal brace are modeled with an elastic perfectly plastic behavior in tension (accounting for the longitudinal reinforcement) and a nonlinear behavior in compression.

Nonlinear dynamic analysis is performed on the structural model for both damage limitation (DL) and significant damage (SD) limit states (LS). Given the complexity of the nonlinear model of the Tower, the choice of the integration method aimed at performing nonlinear dynamic analysis was a key issue. Direct numerical integration approach and Hilber Hughes and Taylor integration method was employed, (Wilson, 2002); 1% mass and stiffness proportional damping was assumed. The latter assumption implies that coefficients of mass and stiffness matrix used to characterize damping matrix have negligible effects; most of structural damping is assumed to be dissipated via hysteretic behavior of elements (Elnashai and Di Sarno, 2008). This latter observation has an experimental counterpart in the damping ratios in Table 1. In fact, along the direction in which tuff infills are present, damping ratio increases. The choice to model explicitly nonlinear behavior of tuff infills for this modern heritage structure is again the proper approach aimed at accounting for the peculiar characteristics of the building.

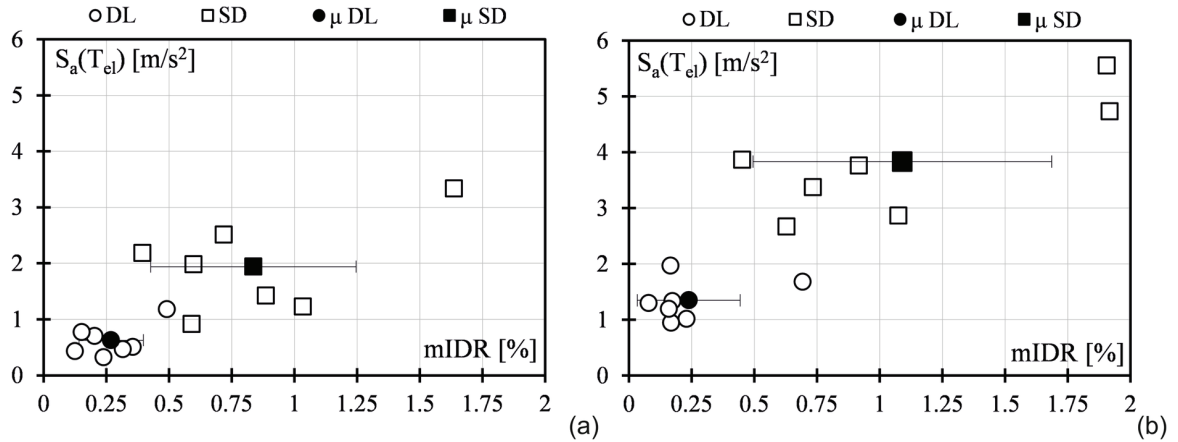
Seismic capacities are evaluated according to EC8 definition for each limit state considered. SD-LS capacity is evaluated as the inter-storey drift (IDR) at the first attainment of 75% ultimate chord rotation capacity in RC elements; while DL-LS capacity is evaluated at the first attainment of yielding rotation capacity in RC elements (see CEN, 2005). It is worth noting that, in the case of DL-LS, capacity evaluation of the model could have been assumed according to a different criterion since infills' contribution has been explicitly modeled. One of the possible approaches is to consider the drift at first attainment of the peak strength in the infill elements (e.g., Dolsek and Fajfar, 2008). On the other hand, given the specific distribution of the infills in the building (along one direction of the building), no difference in capacity is assumed, and it is represented by the first attainment of θ_y in the building. Thus, capacity of the building is ruled by the only RC elements.

Maximum interstorey drift ratio (mIDR) is the engineering demand parameter (EDP) chosen to characterize the seismic response of the Tower. A common way to represent results of nonlinear analyses is to choose the Intensity Measure (IM) and plot analysis results in IM-EDP plane (Figure 9). Elastic spectral acceleration at the fundamental period of the building in the two directions, $S_a(T_{el})$, is the chosen IM, while mIDR is the synthetic demand parameter at the IM value considered. The plot in Figure 9 is a representation of EDPIIM (EDP given IM) as is can be obtained when an EC8 code-conforming nonlinear time-history analysis is carried out.

Comparison of structural response in the two directions in Figure 9 emphasizes the differences in terms of IMs for the two directions of the building; such differences are the results of differences in terms of periods between X direction and Y direction, in which stiffness contribution of the infills affects the dynamic properties of the building. Structural demand is higher in Y direction in the case of SD limit states, while for DL limit states demand is comparable for the two directions.

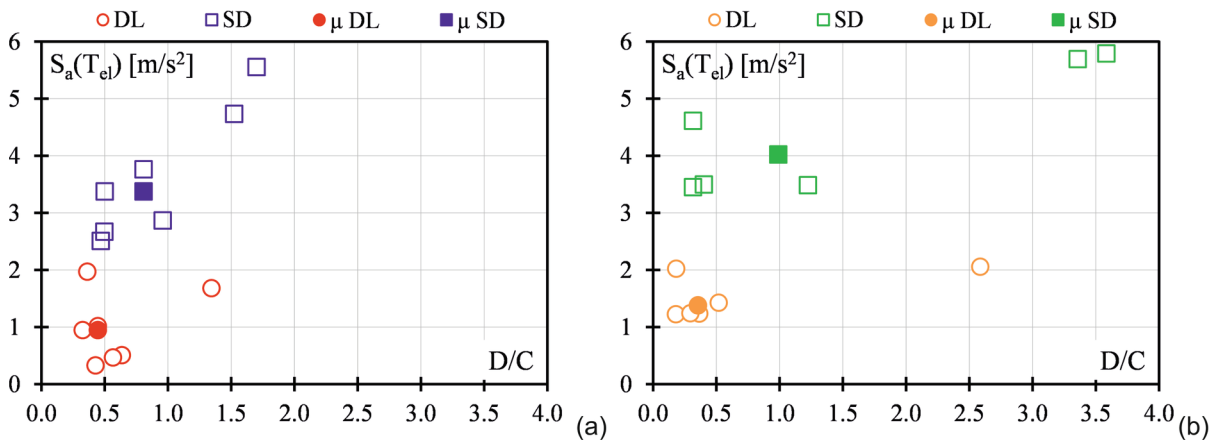
Code-based approaches, and in particular EC8, identify the capacity of buildings with the attainment of conventional limit states in the first element. The IDR characterizing capacity attainment for each limit states, in each direction, can be crossed to obtain maximum D/C of the structure. On the other hand, results are obtained for the two directions of the building, thus performances in each direction

Figure 9. IM-EDP results in X direction (a), and Y direction (b)
(Adapted from De Luca et al., 2014)



need to be properly *combined* aimed at a synthetic global representation of structural performances for the whole structure at each limit states through D/C versus IM plot, as shown in Figure 10. No specific recommendations are available on the way to combine results for the two directions; thus, in Figure 10 two different approaches for the evaluation of whole structural performances are provided. In Figure 10a the maximum D/C are shown as the maximum among X and Y direction for each record (*maximum envelope*) and the IM is that corresponding to the direction in which maximum D/C is attained. In Figure 10b, another combination approach is considered; SRSS of D/C and IM are respectively plotted on the axes. Both performance evaluation approaches provided in Figure 10 emphasizes that the structure can be considered safe for both limit states. In fact, the median D/C ratios (μ) are always lower than 1. Further details of demand and performance concentration at each storey of the building can be found in De Luca et al. (2014).

Figure 10. Performance evaluation in terms of D/C through maximum envelope (a) and through SRSS (b) of X and Y direction results



Nonlinear dynamic analysis performed according to EC8 allows, in some cases, the evaluation of structural fragilities of buildings for the limit states considered. In fact, code-based nonlinear analyses performed with spectrum compatible records can be considered as a cloud analysis (Jalayer et al., 2007; Jalayer et al. 2014). Cloud analysis implements non-linear dynamic analysis results in a (linear) regression-based probabilistic model in log-log plane D/C versus IM. The model can be described according to equations 1 and 2, where Y is the performance variable, in this case D/C of the structure, $\eta_{Y|S_a}$ is the median for Y given S_a (i.e., the IM considered) and $\sigma_{\log Y|S_a}$ is the logarithmic standard deviation for Y given S_a , assumed as constant with S_a (homoscedasticity hypothesis). The lognormal fragility curve is obtained through equation 3, see Jalayer et al. (2014) for further details. In equation 1, $\log a$ is the intercept of the linear function representing $\log(D/C)$ as a function of $\log(S_a)$; while b is the slope of the line in log-log plane.

$$E[\log Y | S_a] = \log \eta_{Y|S_a} = \log a + b \log S_a \quad (1)$$

$$\sigma_{\log Y|S_a} = \sqrt{\frac{\sum_{i=1}^n (\log Y_i - \log \eta_{Y|S_a,i})^2}{n-2}} \quad (2)$$

$$P(Y > 1 | S_a) = P(\log Y > 0 | S_a) = 1 - \Phi\left(\frac{-\log \eta_{Y|S_a}}{\sigma_{\log Y|S_a}}\right) = \Phi\left(\frac{\log \eta_{Y|S_a}}{\sigma_{\log Y|S_a}}\right) \quad (3)$$

Figure 11 and 12 shows the linear regression for DL-LS and SD-LS in the cases of maximum D/C envelope and SRSS D/C combination, corresponding respectively to results provided in Figure 10a and 10b. Figure 11 and 12 show also a , b , and σ_{\log} for the evaluation of the corresponding lognormal fragility curves.

Figure 11. Linear regression and estimation of cloud parameters for DL (a) and SD (b) in the case of maximum envelope of D/C (see Figure 10a)

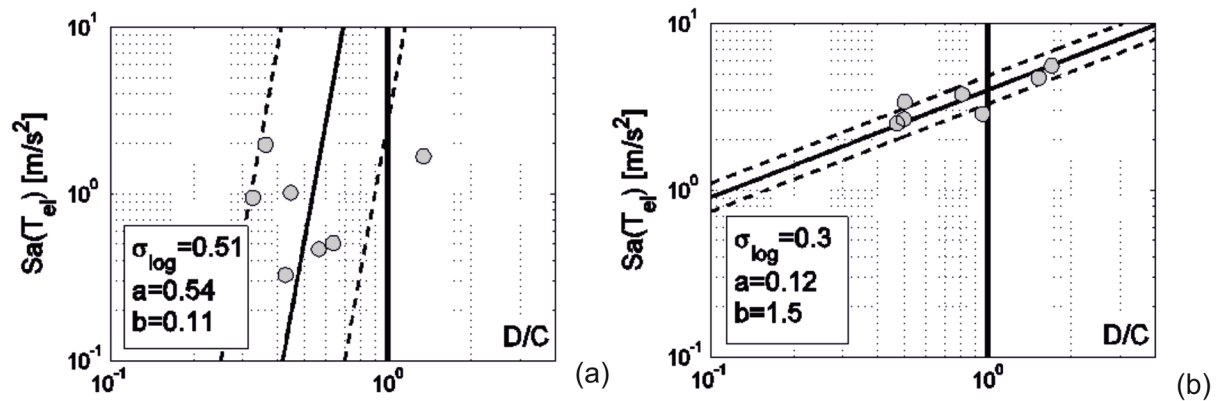
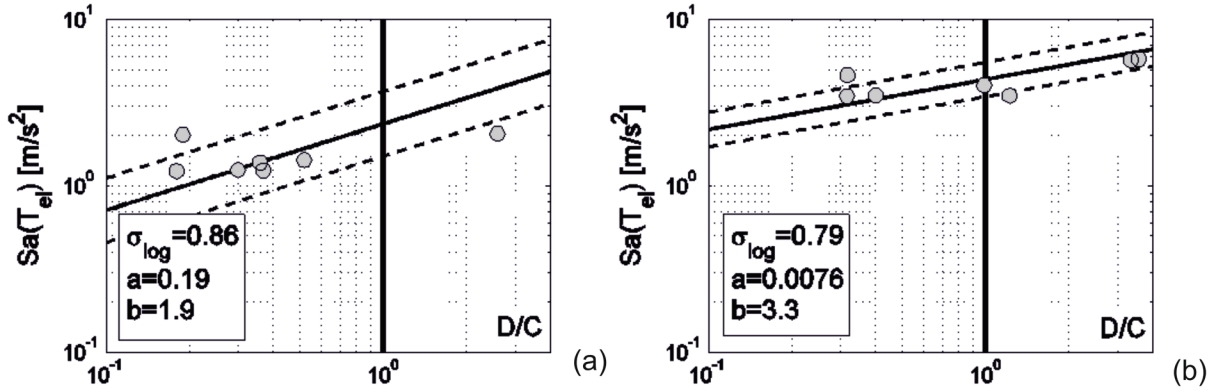


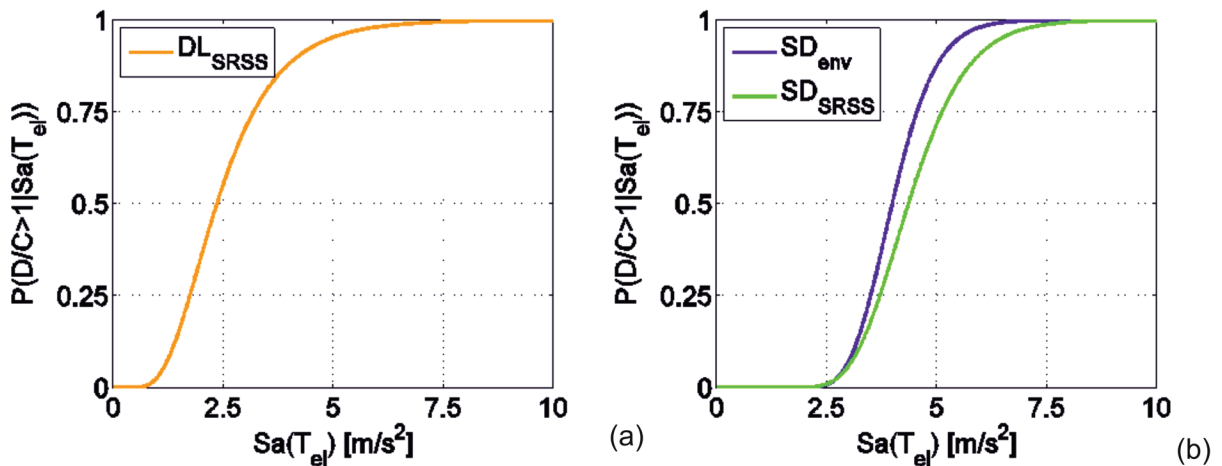
Figure 12. Linear regression and estimation of cloud parameters for DL (a) and SD (b) in the case of SRSS of D/C (see Figure 10b)



Regression parameters in Figure 11 and 12 emphasize how the code-based spectral compatible selection is not always suitable for the evaluation of fragility curves. In fact, it should be noted that cases in which b has small value (e.g., smaller than 1) cannot provide a fragility curve. Maximum envelope D/C ratios for DL (see Figure 11a) provide a linear regression that is clearly not informative of the range of S_a in which D/C is equal to 1; the solid black regression line does not intersect with unity solid vertical line in IM ranges investigated by the analyses performed. The same lack of significance of the results is found also in the case of SRSS combination for DL. In this latter case, the fragility curve can be obtained but it is not informative enough given the large value obtained for σ_{log} (equal to 0.86).

Once a , b , and σ_{log} are obtained, and they can be considered significant, fragility curves expressing the probability that D/C exceeds unity at a given S_a (i.e., IM) level for the specific LS considered can be plot as shown in Figure 13.

Figure 13. Fragility curves for DL and SD in the cases in which cloud analysis carried out b values higher than 1



The SD-LS case allowed carrying out two fragility curves, one for the maximum D/C envelope case, and another for the SRSS cases, see Figure 13b. The S_a ranges considered for SD-LS provided an accurate description of D/C ratios close to unity. Comparison of σ_{\log} for the two curves in Figure 13b, respectively equal to 0.3 and 0.79, emphasizes that the only fragility resulting from the regression in Figure 11b can be considered for sufficiently reliable performance estimation purposes.

CONCLUSION AND PERSPECTIVES

Management, maintenance and preservation of heritage structures usually result in a very complex task. Old design and construction techniques, unique structural schemes, out of sight design and construction defects or even damages due to extreme events often affect this kind of structures.

The assessment framework provided for ordinary structures by codes (e.g., EC8) needs a critical overview and the role played by engineering judgment becomes crucial. Each heritage structure is a specific case and structural modeling needs to take into account for peculiar characteristics of this kind of buildings (e.g., nonlinear modeling of nonstructural elements such as infills). On the other hand, often, limited destructive investigations are allowed for the structural characterization. Therefore, the availability of experimental estimates of the modal properties becomes relevant for structural and seismic assessment purposes. Proper analysis methodologies have to be selected in a balance between most accurate analysis approaches and feasibility of them. In some cases, the choice of nonlinear analysis methodologies carried out according to code prescriptions, also allows the employment of structural reliability methodologies (e.g., cloud analysis) for a better characterization of uncertainties.

All the above issues have been analyzed through a test-bed modern heritage building, the Tower of the Nations, for which critical aspects are emphasized and feasible approaches are pursued for the final aim of seismic structural assessment.

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KEY TERMS AND DEFINITIONS

Cloud Analysis: It implements non-linear dynamic analysis results in a (linear) regression-based probabilistic model.

Dynamic Identification: Evaluation of modal properties of a building through the analysis of the response to induced vibration on the structure.

In Situ Inspection: Direct inspection of structural details of a building (e.g., reinforcement) through destructive techniques (e.g., removing concrete cover).

In Situ Testing: Laboratory testing of materials through specimens taken directly from the structure (e.g., concrete specimens, reinforcement bar specimens).

Modern Heritage Structure: Structure realized with building material currently used also nowadays (e.g., reinforced concrete, steel) that represents a heritage since long time has passed from its realization and it provides information of construction practice of a previous age.

APPENDIX: LIST OF ABBREVIATIONS

- EC8: Eurocode 8
- RC: Reinforced Concrete
- D/C: Demand over Capacity ratio
- KL2: Normal knowledge level defined according Eurocode 8 part 3
- OMA: Operational Modal Analysis
- SRSS: Square Root of Sum of Squares
- CQC: Complete Quadratic Combination
- E_c : Concrete Young modulus
- E_w : Tuff Young modulus
- G_w : Tuff shear modulus
- τ_0 : Tuff shear cracking strength
- LS: Limit State
- DL: Damage Limitation
- SD: Significant Damage
- IM: Intensity Measure
- EDP: Engineering Demand Parameter
- IDR: Inter-Storey Drift
- mIDR: maximum Inter-Storey Drift
- EDP|IM: EDP given IM, relation resulting from structural analysis
- μ : median
- S_a , $S_a(Tel)$: Spectral acceleration at the fundamental period of the structure, assumed as IM in this study
- Y : performance metric, in this study assumed equal to D/C
- $\eta_{Y|S_a}$: median estimate of Y at a given S_a value obtained through linear regression
- $\sigma_{\log Y|S_a}$, σ_{\log} : constant standard deviation of logarithms of Y at a given S_a (homoscedasticity assumption).